ATTACHMENT D

Converse Consultants

Geotechnical Engineering, Environmental & Groundwater Science, Inspection & Testing Services

August 28, 2007

System Operations Services, Inc.
c/o Mr. John C. Pings
Poseidon Environmental Construction, Inc.
1941 New York Drive
Altadena, California 90010-2241

Subject: PORTLAND CEMENT CONCRETE PAVEMENT SECTION
Atlas Iron & Metals, Inc.
10019 South Alameda Street
Unincorporated County of Los Angeles, California
Converse Project No. 07-31-252-01

Gentlemen:

In response to your request, Converse Consultants (Converse) is presenting recommendations for Portland cement concrete pavement for the project at Atlas Iron & Metals, Inc located in the unincorporated area of the County of Los Angeles, California.

Samples of the subgrade soil underlying the existing fill material was obtained as part of the field investigation for the adjacent property line wall. This material is classified as Sandy Silt (ML). Laboratory testing was performed on the sample to determine the R-value of the material in accordance with ASTM Test Method D-2844. The results of the testing indicate that the on-site soils have an R-value of 20.

It is our understanding that the subject pavement will be used for the storage of empty shipping containers. Forklifts will be used to move the containers within the area. We calculated the traffic loading conditions was based upon approximate five repetitions of a rubber tired forklift with container per week and a given maximum weight of the forklift with empty container of 16 kips. When carrying an empty container we have assumed that 85 percent of the weight of the forklift and container is on the front (drive) wheels of the forklift. The maximum wheel loading for this condition is 6.8 kips.

The equivalent traffic index for the loading calculated in accordance with State of California, Department of Transportation (Caltrans) procedures is 3.4 (3.5 used in pavement calculation).

Based upon the traffic index of 3.5, the following Portland cement concrete pavement section was determined using a design chart. The design chart was developed based upon information contained in "Portland Cement Concrete Pavement Design for Light, Medium and Heavy Traffic" published by the Portland Cement Concrete Association in

1981. The R-value of the combined base material and the compacted subgrade were determined in accordance with Caltrans' Highway Design Manual. A copy of the calculations and the design chart are attached to this letter/report.

Portland cement concrete thickness	6 inches
Aggregate base thickness	8 inches
Compacted subgrade soil	12 inches

The Portland cement concrete should have a minimum modulus of rupture of 550 pounds per square inch (psi).

Base material should be Crushed Miscellaneous Base and should be compacted to at least 95 percent of the ASTM D1557 laboratory maximum density. Crushed Miscellaneous Base materials should conform to Sections 200-2.2 of the *Standard Specifications for Public Works Construction* (Greenbook), 2003 Edition. To reduce the potential for premature pavement distress, it is important that final pavement grade be designed such that ponding on or adjacent to the pavements is avoided. Pavement runoff should be directed to a suitable non-erosion drainage device.

In the proposed pavement area, the upper twelve inches of subgrade soils should be over-excavated as needed, scarified, moisture-conditioned, and compacted to a minimum of 90 percent of the ASTM D1557 laboratory maximum density at a moisture content between optimum and 2 percent of optimum to a minimum depth of 12 inches. Moisture content of the subgrade soil should be maintained until the base material is placed and compacted.

We trust that this information is adequate for your current needs. Please contact the undersigned at (626) 930-1233, if you have any questions or wish to discuss our proposal in greater detail.

CONVERSE CONSULTANTS

//original signed by//

J. Stanley Schweitzer, GE
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
J. STANLEY SCHWEITZER
G.E. NO. 758
EXP. 3-31-08
GEOTECHNICAL
STATE OF CALIFORNIA

Dist.: 2/addressee

Encl.: Calculation Sheet 1 of 1
P.C.C. Pavement Design Chart for City Count Roads

JSS/dlr

BY JSS DATE 27Aug CLIENT Atlas Metals SHEET NO. 1 OF 1
CHKD. BY ____ DATE ____ PROJECT NO. 07-31-252-01
PROJECT Atlas Metals

Traffic Index Calculations

Given = Traffic - Fork Lift w/ 5 rep/week
Assume Forklift & Load = 16k w/ 85% on Drive Wheels

Wheel Load = $.85(16k)/_2$ = 6.8 kips per Drive Wheel

$EWL_{/rep} = (6.8/5)^{4.2} = 3.64$

Total Rep = 5 rep/week · 52 weeks/yr · 20 yr = 5200 rep

$TI = 6.7\left(\frac{EWL}{10^6}\right)^{.119}$

$= 6.7\left(3.64 \times 5200/_{10^6}\right)^{.119} = 3.4$ use 3.5

PCC Pavement Thickness

$R_{value_{subgrade}}$ = 20 from Lab
Assume 60% increase for 8 in AB per caltrans
Rvalue w/ Base = 32. use 30

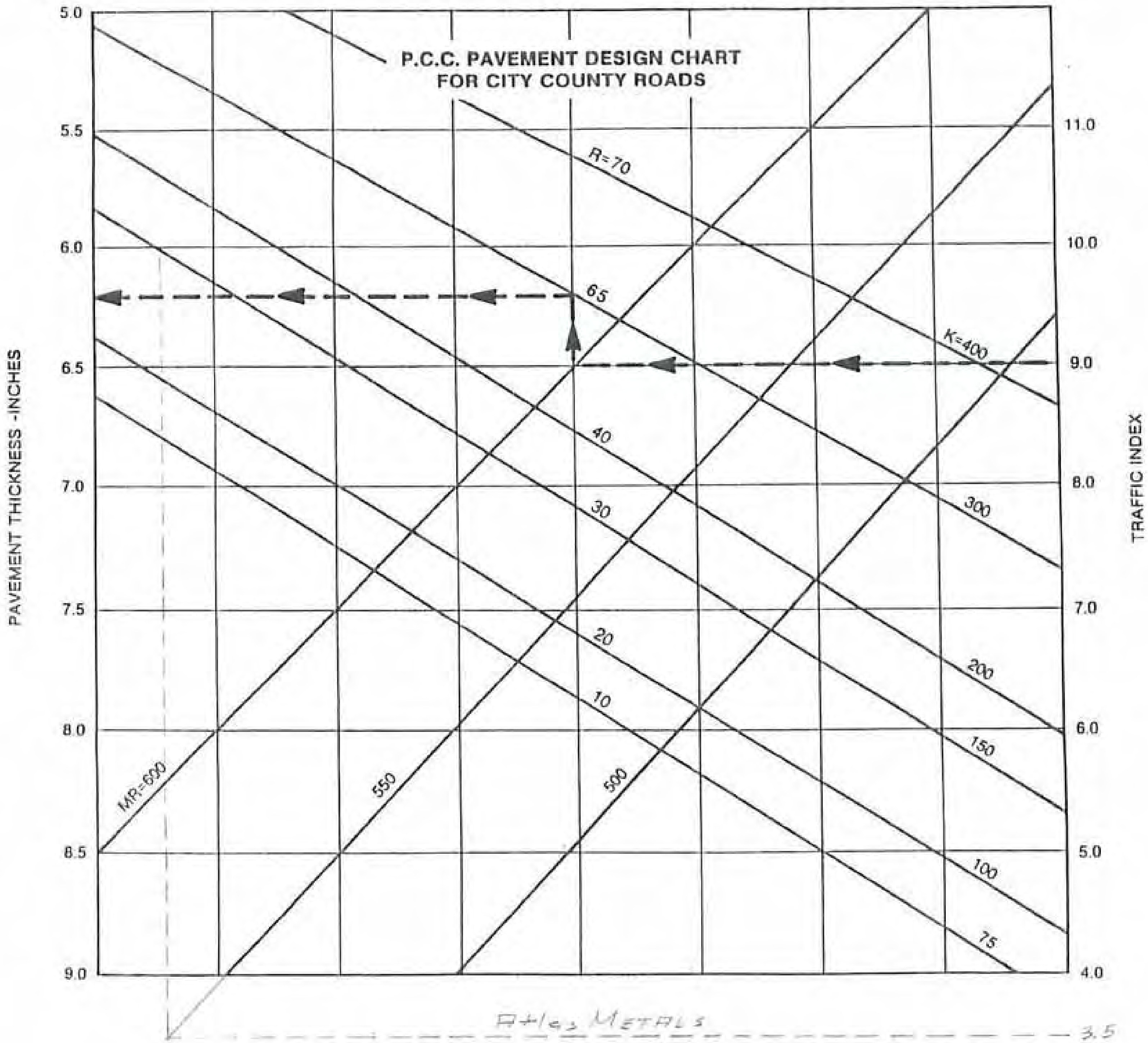
$t_{From\ Nomograph*}$ = 6.0 inches with M_r = 550 psi

* Reference: Portland Cement Assoc., 1981, "Portland Cement Concrete Pavement Design for light, Medium & Heavy Traffic

Converse Consultants

SIGNED ____
REG. NO. ____
SHEET NO.

L-5-10/81



Converse Cons...
Project ...

ATTACHMENT E

GEOTECHNICAL INVESTIGATION REPORT
Property Line Wall
Atlas Iron & Metals, Inc.
10019 South Alameda Street
Unincorporated County of Los Angeles, California

Converse Project No. 07-31-252-01

August 31, 2007

PREPARED FOR

System Operation Services, Inc.
2140 Shattuck, Avenue, 2nd floor
Berkeley, CA 94704

Converse Consultants

Geotechnical Engineering, Environmental & Groundwater Science, Inspection & Testing Services

August 31, 2007

Dr. Larry Russell, PE
System Operation Services, Inc.
2140 Shattuck, Avenue, 2nd floor
Berkeley, CA 94704

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Property Line Wall
Atlas Iron & Metals, Inc.
10019 South Alameda Street
Unincorporated County of Los Angeles, California
Converse Project No. 07-31-252-01

Dear Dr. Russell:

We are pleased to present this geotechnical investigation report for the proposed New Property Line Wall at Atlas Iron & Metals, Inc in the Walnut Park Area of Unincorporated Los Angeles County, California. This report was prepared in accordance with our July 17, 2007, proposal and your subsequent authorization.

Based on our field investigation, laboratory testing, geologic evaluation and geotechnical analysis, the site is suitable from a geotechnical standpoint for the proposed Property Line Wall Project provided our conclusions and recommendations are implemented during design and construction. The findings of the investigation and recommendations for the design and construction of the structure are presented in the attached report and are summarized in the Executive Summary Section following this letter. Recommendations related to the design and construction of Portland cement concrete pavement adjacent to the wall were previously presented in a separate letter/report.

Thank you for this opportunity to be of service. If you have any questions, or if we can be of additional service, please do not hesitate to contact us.

CONVERSE CONSULTANTS

//original signed by//

J. Stanley Schweitzer, GE 758
Senior Geotechnical Engineer

Dist: 4/Addressee
1/ Poseidon Environmental Construction, Inc, Attn.: Mr. John C. Pings via e-mail

JSS/WHC/dlr

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EXECUTIVE SUMMARY

The following is a summary of our Geotechnical Investigation, Findings, Conclusions, and Recommendations, as presented in the body of this report. This summary is presented for the cursory review of the investigation report and may not be adequate for other purposes. The summary should not be used separately for design and/or construction. Please refer to the appropriate sections of the report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The subject site is considered suitable from a geotechnical engineering viewpoint for the proposed property line wall provided that the recommendations presented in the attached report are incorporated into the design and construction.
- The field exploration for the geotechnical investigation consisted of drilling 3 exploratory borings to depths varying from approximately 2.5 to 51.5 feet below the existing ground surface on August 9, 2007. Subsurface conditions encountered in the borings were logged and classified in the field by visual/manual examination, in accordance with the Unified Soil Classification System.
- Laboratory testing of soil samples collected during the geotechnical investigation included moisture and density determinations, compaction, direct-shear strength, consolidation, sieve and hydrometer analysis, expansion index, pH, minimum electrical resistivity, soluble sulfate, and chloride concentration testing.
- The site is not within a currently designated State of California Fault Hazard Zone. The nearest fault is the Newport-Inglewood, located approximately 5.3 miles southwesterly of the subject site. Due to the close proximity of the site to the fault, there is a high probability of strong shaking at the site during a strong seismic event on the Newport-Inglewood Fault.
- Groundwater was encountered during this investigation at a depth of approximately 43.0 feet below the existing ground surface. However, the site is in an area of mapped potential liquefaction based on a historical high groundwater surface on the order of ten feet below the ground surface. We have performed analysis for both groundwater levels and have concluded that there will be significant settlement (up to approximately 14 inches) of the ground surface in the area of the wall during a major earthquake if the groundwater is at near historical high conditions. With the groundwater at the current level, the anticipated settlement of the ground surface in the area of the wall during a major earthquake will be on the order of three inches or less.
- Evidence of existing fill soils was encountered in two of the borings for this investigation. The fill is not considered suitable for support of the proposed wall. It is recommended that wall footings extend through the fill into the native soils beneath the fill.

- The underlying alluvial soils encountered during this investigation are in general silty sands, sandy silts and fine sands.
- The on-site soils are expected, based upon classification and laboratory testing, to possess a low expansion potential, as defined by the Los Angeles County Building Code. Special design and/or construction for expansive soil conditions on this project have been incorporated into the earthwork and foundation design recommendation.
- Surface drainage should be sloped away from the structure. Ponding of surface water should not be allowed adjacent to the structure.
- Temporary construction slopes, greater than four feet in height, should be sloped or shored in accordance with the requirements of CAL-OSHA.

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Property Line Wall
Atlas Iron & Metals, Inc.

Converse Project No. 07-31-252-01

1.0 INTRODUCTION

This report presents results of a geotechnical investigation performed by Converse Consultants (Converse) for the proposed property line wall at Atlas Iron and Metals in the Walnut Park Area of Unincorporated Los Angeles County, California. The purposes of this investigation were to determine the nature and engineering properties of the earth materials at this site, and to provide geotechnical recommendations for design and construction of the proposed school expansion project.

This report is for the proposed property line wall described herein, and is intended for use by System Operation Services, Inc. and its design professionals. Since this report is intended for use by the designer(s), it should be recognized that it is impossible to include all construction details in this report at this phase in the project. Additional consultation may be prudent to interpret these findings for contractors, or possibly refine these recommendations based upon the final design and actual conditions encountered during construction.

Recommendations for the design and construction of Portland cement concrete pavement for the area along the east side of the subject wall were previously presented in a separate letter/report entitled *"Portland Cement Concrete Pavement Section" dated August 28, 2007.*

2.0 PROJECT/SITE DESCRIPTION

Atlas Metals is located westerly side of South Alameda Street, just southerly of the intersection of Alameda Street and Tweedy Boulevard in the Walnut Park Area of Unincorporated Los Angeles County, California. The general location of the Atlas Metals is shown on Figure No. 1, *"Site Location Map".*

The proposed wall will be located along the westerly property line. Los Angeles Unified School District's Jordan High School is located just beyond the property line. Light to medium industrial facilities are located to the north and south of the Atlas Metals property.

The property is currently being used as a metal recycling facility with the southerly half of the westerly portion of the site paved with Portland cement concrete and used for storage of shipping containers and equipment. The northerly half of the site adjacent of the westerly property line has recently been excavated to approximately one foot below the elevation of the top of pavement on the southerly portion and covered with plastic sheeting. It is understood that following construction of the proposed wall the area adjacent to the wall will be used for storage of empty shipping containers only.

The subject project will consist of the design and construction of a concrete property line wall. The wall will be approximately 280 feet long and will be constructed of precast concrete panels supported by cast-in-drilled-hole concrete piles and grade beam.

In the absence of structural loading information, we have assumed for the purpose of this proposal that maximum continuous wall load will be on the order of 2 kips per lineal foot.

3.0 SCOPE OF WORK

The scope of geotechnical services performed for this project included exploratory borings, geotechnical laboratory testing of soil samples, geotechnical engineering analyses, and preparation of this written report. This report did *not* include an evaluation of the potential for soil and/or groundwater contamination at this site. The scope of work for this investigation included the following:

- Field exploration consisted of drilling 3 exploratory borings (BH-1 through BH-3) to depths varying from 2.5 to 51.5 feet below the existing ground surface at the locations shown on Figure No. 2, *"Site Plan and Approximate Boring Locations Plan"*. Subsurface conditions encountered in the borings were continuously logged and classified in the field by visual/manual examination in accordance with the Unified Soil Classification System. Field exploration procedures and boring logs are presented in Appendix A, *Field Exploration.*

- Laboratory testing included moisture and density determinations, compaction, direct-shear strength, Expansion Index, , sieve and hydrometer analysis, pH, resistivity, soluble sulfate, and chloride concentration testing. Descriptions of the individual tests and test results are presented in Appendix B, *Laboratory Test Program.*

- Engineering analyses and evaluation of results of the field exploration and laboratory testing were performed to develop design and construction recommendations for the proposed school buildings. Findings and recommendations are documented in this written report.

4.0 SUBSURFACE CONDITIONS

Portland cement concrete pavement on the order of eight inches in thickness was encountered at two of the exploratory boring locations. This pavement covers the southerly half of the site adjacent to the westerly property line.

Evidence of old fill containing significant amounts of debris was encountered at the northerly two boring locations. At the boring location (BH-1)at approximately the center of westerly property the depth of fill encountered in the boring was approximately five feet. Boring BH-3 was drilled near the northerly end of the west property line. At this location, the equipment could not drill through the fill due to the amount of debris located within the fill. The maximum depth of this boring was on the order of 2.5 feet below the ground surface.

Native soils encountered below the pavement and/or fill in the borings are predominately fine sands, silty sands and sandy silts. These natural soils are generally medium dense or firm. A large portion of the soils encountered above the groundwater were very moist to wet.

Groundwater was encountered at a depth of approximately 43 feet below the ground surface. Historical high groundwater surface is approximately ten feet below the existing ground surface.

Based on the results of subsurface exploration and experience, variations in the continuity and depth of subsurface conditions should be anticipated. Care should be exercised in interpolating or extrapolating subsurface conditions between or beyond borings. Fill depths should be expected to vary between borings.

5.0 CONCLUSIONS

The following conclusions are based on the results of the field investigation, laboratory testing and our understanding of the scope of the project.

- The site is suitable from a geotechnical viewpoint for the proposed construction of a property line wall, provided that the recommendations presented in this report are incorporated into the design and construction of the project.
- Evidence of undocumented fill was encountered in two borings drilled. The recommended pile foundation will have to extend through the fill into the underlying native soil.
- Groundwater was encountered at a depth of 43 feet below the ground surface. As a result of the current depth to groundwater, it is not expected to be encountered during the construction of the proposed building.
- There are no active faults projecting toward or extending across the proposed site. The site is not located within a currently designated State of California Earthquake Fault Hazard Zone. However, due to the close proximity of the site to the Newport-Inglewood fault and other nearby fault zones, very strong shaking could result from a major seismic event on this fault.
- Site soils appear to be susceptible to liquefaction under earthquake ground shaking, However, the affects of possible liquefaction are expected to be limited to settlement of the ground surface.
- Site soils should be able to be excavated with conventional heavy-duty earthmoving equipment.
- Based upon the soil classification and laboratory testing performed, a very low, as defined in Table 18-1-B of the 2002 Los Angeles County Building Code (LACBC) has been assumed. Special design and/or construction for expansive soil conditions have been incorporated into the earthwork and foundation design recommendation.

County 110 Statement

Based upon the results of this investigation, we have concluded that the site and proposed development will be safe from landsliding, settlement or slippage, provided that the recommendations herein are incorporated into the design and construction. The pile foundation recommended will is designed to mitigate the adverse affects of the liquefaction. Furthermore, the proposed property line wall will not adversely affect the stability of property outside the limits of proposed construction.

6.0 SEISMICITY

6.1 General

The site, as is all of Southern California, is located within a seismically active area. However, it is not within a currently designed Fault Hazard Zone, but is located approximately 5.3 miles (6.9 kilometers) northeasterly of the surface expression of the Newport-Inglewood Fault-Rupture Hazard Zone. Accordingly, strong ground shaking due to seismic activity is anticipated at this site. The provisions of the California Building Code (CBC 2001 or 2007 Edition) and the Structural Engineers Association of California (SEAOC) guidelines are considered appropriate for design of the facility.

6.2 2001 CBC Near Source Parameters

Based on the available site data, it is our opinion that Soil Profile Type S_D, as defined in Section 1636 of the 2001 CBC, is appropriate for the site.

The nearest known fault to the site is the Newport-Inglewood fault. Based on Tables 16-S and 16-T, the recommended values of near-source factors N_a and N_v occur for the Newport-Inglewood Fault with a distance of 2.3 kilometers. Accordingly, the value of N_a is 1.0 and the value of N_v is 1.2. Using a Seismic Zone Factor of 0.4, seismic coefficients C_a and C_v are 0.44 and 0.74, respectively.

Faults within 20 km of the site are given in Table 1, *2001 CBC Seismic Design Parameters*. Fault information was taken from California Geologic Survey – 2002 California Probabilistic Seismic Hazard Maps. According to Tables 16-S and 16-T of the 2001 CBC, faults more than 15 km from a site do not affect near-source factors. All faults closer than 15 km are Seismic Source Type B faults.

Table No. 1
2001 CBC Seismic Design Parameters

Fault	Moment Magnitude M_w	Slip Rate Mm/year	Closest* Distance to Site (km/mi)	Seismic Source Type
Newport-Inglewood	7.1	1.0	6.9/5.3	B
Hollywood	6.4	1.0	19.1/11.8	B
Raymond	6.5	1.5	19.5/12.1	B
Palos Verdes	7.3	3.0	20.0/12.4	B
Elsinore-Whittier	6.8	2.5	20.0/12.4	A

*Closest distance to surface projection of the rupture area.

6.3 2007 CBC (2006 IBC) Seismic Parameters

Based on the results of our borings, and laboratory testing, and in accordance with Table 1613.5.2 of the California Building Code (2007 CBC) and the International Building Code (2006 IBC) the site should be considered as Site Class D. The Site Coefficients F_a, F_v, defined from Tables 1613.5.3(1), and 1613.5.3(2), as following:

F_a=1.0 F_v=1.5

The mapped maximum considered earthquake spectral response acceleration at short period, S_s, and at 1-second period, S_1, for a site Class B is determined at 2006 IBC, Figures 1613.5(3) and 1613.5(4), as following:

S_s=1.583 S_1= 0.615

The mapped maximum considered earthquake spectral response acceleration at short period, S_s, and at 1-second period, S_1, for the subject Class D site is as following:

$$S_{MS} = F_a\, S_s = 1.583$$
$$S_{M1} = F_v\, S_1 = 0.922$$

6.4 Liquefaction Evaluation

Liquefaction is the sudden decrease in shearing strength of cohesionless soil due to vibration. During dynamic or cyclic shaking, the soil mass is distorted, and interparticulate stresses are transferred from the sand grains to the pore water. When the pore water pressure increases to the point that the interparticulate effective stresses are reduced to zero, the soil behaves temporarily as a viscous fluid (liquefaction) and, consequently, loses its capacity to support the structures founded thereon.

Liquefaction potential has been found to be the greatest where the groundwater level and loose sands occur within a depth of about 50 feet or less. The potential for liquefaction decreases with increasing grain size and clay and gravel content, but increases as the ground acceleration and duration of shaking increase.

Groundwater was encountered at a depth of 43 feet below the existing ground surface. Historical high groundwater level as presented in the Seismic Hazard Report for the area is on the order of ten feet below the existing ground surface.

As indicated in the Logs of Borings, the site soils are predominately sandy silts, silty sand and fine sand. These soils are considered to be liquefiable during a major earthquake.

Liquefaction analysis has been performed using the computer program "*LiquefyPro*" by Civil Tech Software. Although the existing groundwater level is at approximately 43 feet below the ground surface, the analysis was performed assuming that the ground water was at the historical high level (ten feet below the ground surface). The results of this analysis indicate that essentially all of the soil below the groundwater and above a depth of 50 feet will have the potential forliquefaction during the Design Based Earthquake (DBE). Due to the relatively level ground surface on and adjacent to the westerly property line, it is expected that the affects of liquefaction will be limited to settlement. No permanent lateral movement of the ground is expected.

The results of the liquefaction analysis indicate that with groundwater at historical high conditions, the expected maximum settlement will be on the order of 13.8 inches. With the groundwater at the existing level (approximately 43 feet below the ground surface) the settlement from liquefaction is expected to be on the order of 1.3 inches. The soils above the groundwater are also expected to settle. The anticipated settlement of the soils above the groundwater is on the order of 1.4 inches for a total maximum settlement at the ground surface of approximately 2.7 inches. Differential settlement along the wall may be as much one half of the total settlement. A more detailed discussion of this analysis and the results of the analysis are presented in Appendix D, *Liquefaction Analysis/Seismically-Induced Ground Settlement.*

Due to the relatively large amounts of settlement anticipated during the DBE, damage to the wall could occur. Support of the wall on a pile foundation designed and constructed in accordance with the recommendations presented in this report will minimize the potential for damage to the wall from seismically induced settlement. The pile capacities presented herein have been significantly reduced from those calculated for static conditions in order to increase the support during the design earthquake. The depths that the liquefaction is expected to occur, indicates that remedial grading along the alignment of the proposed wall will not significantly reduce the settlement.

6.5 Secondary Seismic Effects

In addition to ground shaking and liquefaction, secondary effects of seismic activity that could impact the project site include surface fault rupture, differential settlement of the structure, ground lurching, land sliding, lateral spreading, earthquake-induced flooding, seiches, and Tsunamis. The results of a site-specific evaluation of the potential for these secondary effects affecting the project site are presented below:

- Surface Fault Rupture: The project site is located approximately 5.3 miles from the Newport-Inglewood fault, which is the nearest known fault to the site. As a result, the potential for surface rupture resulting from the movement of this fault or other nearby faults, although not known with certainty, is considered to be low.

- Landslides: The potential for seismically induced landslides and/or other types of slope failures, such as lateral spreading on or adjacent to slope surfaces, adversely affecting the site is considered to be very low, due to the absence of slopes on or adjacent to the site.

- Lateral Spreading: Seismically induced lateral spreading involves primarily lateral movement of earth materials due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. The topography at the project site and in the immediate vicinity of the site is relatively flat. Under these circumstances, the potential for lateral spreading at the subject site is considered very low.

- Seismically Induced Settlement: Seismically induced settlement occurs as the result of loose, sandy soil densifying during strong shaking from an earthquake. Review of the subsurface information obtained from this investigation indicates that the sandy soils below the proposed wall structure will consolidate during a major earthquake. Assuming that that groundwater is at current level or lower at the time of the earthquake the consolidation will result in approximately 2.7 inches of settlement of the ground surface. Our analysis indicates that the consolidation will occur uniformly over the soils above the groundwater level. We estimate that maximum differential settlement will be on the order of 1/2 of the total settlement. Recommendations for foundation design will minimize the potential for wall damage resulting from this settlement. A more detailed discussion of this analysis and the results of the analysis are presented in Appendix D, *Liquefaction Analysis/Seismically-Induced Ground Settlement.*

- Tsunamis/Seiches: Tsunamis and seiches are large seismic generated waves in the ocean (Tsunamis) or large enclosed bodies of water (Seiches). Based upon the distance of the site from the ocean and/or lakes and/or reservoirs, the potential of Tsunamis and/or Seiches affecting the site are considered to be very low.

- Earthquake-Induced Flooding: This is flooding caused by failure of dams or other water-retaining structures up gradient of the site as a result of an earthquake. Review of the area adjacent to the site indicates that there are no significant up gradient lakes or reservoirs with the potential of flooding the site.

7.0 DESIGN RECOMMENDATIONS

7.1 General

The proposed wall may be supported on cast-in-drilled-hole pile foundation or conventional continuous footings bearing on completely on undisturbed native soils.

Calculations indicate that the site will be subject to severe settlement as a result of liquefaction and/or seismically induced settlement of the native soils below the site. This settlement is expected to cause damage to the wall and existing adjacent structures. The recommendations presented herein have been developed to reduce the damage to the wall and to minimize the potential of wall collapse.

In the subsections below, design recommendations for earthwork, foundations, slabs-on-grade, and corrosion and chemical attack resistance are provided. Construction considerations, such as temporary excavations, are discussed in the Construction Considerations section presented later in this report.

7.2 Earthwork

Earthwork is expected to consist of excavation for the cast-in-drilled-hole piles and the connecting grade beam. Remedial grading below or adjacent to the foundation is not anticipated. If remedial grading becomes necessary the limits of the grading should be set based upon the actual conditions encountered during construction. Earthwork should be performed in accordance with recommendations are presented in Appendix C, *Recommended Earthwork Specifications*.

7.3 Pile Foundation

Downward capacity of cast-in-drilled-hole piles may be calculated an average friction value of 300 pounds per square foot for the portion of the pile that extends below the bottom of the existing fill. Pile capacities for the above skin friction value are based upon geotechnical considerations only and actual pile capacities maybe limited by structural considerations such as the strength and rigidity of the reinforced concrete pile as a structural element. Due to the potential settlement of the existing fill and/or surcharging the existing retaining wall, all support of the piles derived from the soil above the existing retaining wall foundation should be ignored in the calculation of downward capacities for support of long-term dead and live loads. The exact depth of the fill at each pile location should be determined in the field during construction. For preliminary design the depth of fill should be assumed to be at least five feet below the existing ground surface.

The piles should be interconnected with a structural grade beam in order to reduce the potential for wall collapse and damage to the wall from settlement of the underlying soils during an earthquake (liquefaction and seismic induced settlement). The grade beam should be a minimum of 18 inches wide and 24 inches deep.

The minimum embedment of piles into native soil below the bottom of the existing retaining wall footing should be ten feet.

In order to eliminate reductions in capacities and problems in construction, the minimum pile spacing should be 3.0 diameters on center.

For design of support of short duration wind and/or seismic loading, downward capacities derived from the above skin friction may be increased by 20 percent. This increase in capacities takes into account the short-term support of the fill and the decrease in soil capacity due to liquefaction of the underlying native soils.

Short term up lift capacities may be assumed to be equal to half the downward friction capacities.

Settlement of piles designed and constructed in accordance with the recommendations presented herein is estimated to be on the order of ½ inch.

Lateral resistance for piles may be assumed to be provided by passive pressure acting on the piles embedded into native soil. The allowable passive pressure for piles spaced at least 3 diameters on center may be taken as 350 psf on the pile per foot of depth, measured below the bottom of existing fill. The allowable maximum passive resistance should not exceed 3,500 psf. It should be noted that the above values for passive earth pressure given for the design of piles have been adjusted for potential arching between piles and no additional increases for arching should be assumed.

7.4 Corrosivity and Chemical Attack

In order to determine the potential affects of the soil on concrete and buried metal pipes, resistivity, pH, soluble chloride and soluble sulfate test results were performed on a portion of a bulk soil sample of the near surface soils recovered at the site, and the results are presented below and in Appendix B, *Laboratory Testing Program*.

Sulfate concentration of 0.006 percent by weight in the sample tested were measured. These sulfate concentrations are defined as a negligible concentration by Table 19-A-3 of the CBC (2001 Edition). As a result, special sulfate-resisting concrete is not currently considered necessary for this project. However, additional testing during construction prior to the placement of footings should be performed to confirm this condition.

Tests performed on a portion of a bulk sample representative of the near surface indicates that the near surface soils have a chloride content of 215 ppm, and pH of 8.38. Minimum resistivity value of 2500 ohm-centimeters were measured on saturated soil samples. These indicate a moderately corrosive potential for ferrous metals in contact with these soils. Therefore, conventional corrosion mitigation measures are considered appropriate for these potentially corrosive soils, which include the following:

- All steel and wire concrete reinforcement should have at least three inches of concrete cover where cast against soil, unformed.
- Below-grade ferrous metals should be given a high-quality protective coating, such as 18-mil plastic tape, extruded polyethylene, coal-tar enamel, or Portland cement mortar.
- Below-grade metals should be electrically insulated (isolated) from above-grade metals, by means of dielectric fittings in ferrous utilities and/or exposed metal structures breaking grade.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 Temporary Excavations

Temporary slopes may be used during excavations where not constrained by adjacent utilities and structures. Where space is limited due to adjacent facilities and buried utilities to be salvaged and protected, shoring may be required. Recommendations for shoring design can be provided upon request.

Based upon the soils encountered in the borings, it is our opinion that sloped temporary excavations may be cut according to the slope ratios presented in the following table:

Table No. 2
Temporary Excavation Slopes

Maximum Depth of Cut (feet)	Maximum Slope Ratio (horizontal:vertical)
0 - 4	vertical
4 - 10	1 : 1

Slope ratios given above are assumed to be uniform from top to toe of slope. Surfaces exposed in sloped excavations should be kept moist, but not saturated, to retard raveling and sloughing during construction. Adequate provisions should be made to protect slopes from erosion during periods of rainfall. Surcharge loads should not be permitted within a horizontal distance equal to the depth of the cut from the top of slopes. There is the potential that sandy strata may be encountered that will require temporary cut slopes to be less steep than tabulated above. As a result, the excavation slope should be observed on a periodic basis during the excavation of the subterranean portion of the structure, in order to verify soil conditions. Workers entering excavations should be protected from possible caving and raveling soils.

8.2 Temporary Shoring

In lieu of sloped excavations deeper than four feet, the excavations may be shored. Design of shoring systems for support of excavations should be for Type C soils as defined by the State of California, Construction Safety Orders.

8.3 Pile Construction

Pile drilling and concrete placement should be performed in accordance the recommendations presented herein and in Appendix E, *Guide Specifications for Drilled Pile Installation* and the Standards and Specifications of ADSC: An International Association of Foundation Drilling Contractors.

It should be noted that the loose fill and some relatively sandy soils were encountered during this investigation. As a result caving of the sidewalls can be expected during the drilling and construction of the cast-in-drilled-hole piles.

Drilling of pile shafts should be observed by Converse to confirm that piles are extended to the proper depth and that material encountered is similar to that encountered in the borings drilled for this investigation. Pile lengths should be tabulated in the foundation plans based upon the embedment into native soil.

During the field exploration, groundwater was encountered at a depth of 43 feet of the existing ground surface. However, it is not expected that groundwater will be encountered during the drilling of pile shafts. Improper placement of concrete in piles may result in either contaminated and/or weak concrete, or voids in the concrete mass. Placement of concrete should be observed and documented by an inspector familiar with constructing piles.

8.4 Geotechnical Services During Construction

This report has been prepared to aid in the evaluation of the proposed structure and to assist architects and engineers in design of the proposed structure. It is recommended that this office be provided an opportunity to review final design drawings and specifications to determine if the recommendations of this report have been properly implemented.

Foundation recommendations in this report are based on the assumption that all structural foundations will be placed on undisturbed native soils. All foundation excavations should be observed by Converse prior to placement of steel and concrete, to verify that foundation elements are founded on satisfactory materials and that excavations are free of loose and disturbed soils. All structural fill and backfill should be placed and compacted during observation and testing by Converse.

During construction, the geotechnical engineer and/or their authorized representatives are present at the site to provide a source of advice to the client regarding the geotechnical aspects of the project and to observe and test the earthwork performed. Their presence should not be construed as an acceptance of responsibility for the performance of the completed work, since it is the sole responsibility of the contractor performing the work to ensure that it complies with all applicable plans, specifications, ordinances, etc.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and cannot be responsible for other than our own personnel on the site; therefore, the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any recommended actions presented herein to be unsafe.

9.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice for Southern California at this time. We make no other warranty, either expressed or implied. Conclusions and recommendations presented in this report are based on results of this field and laboratory investigation, combined with an interpolation and extrapolation of subsurface conditions between and beyond boring locations. If conditions encountered during construction appear to be different from those assumed in this report, this office should be notified immediately.

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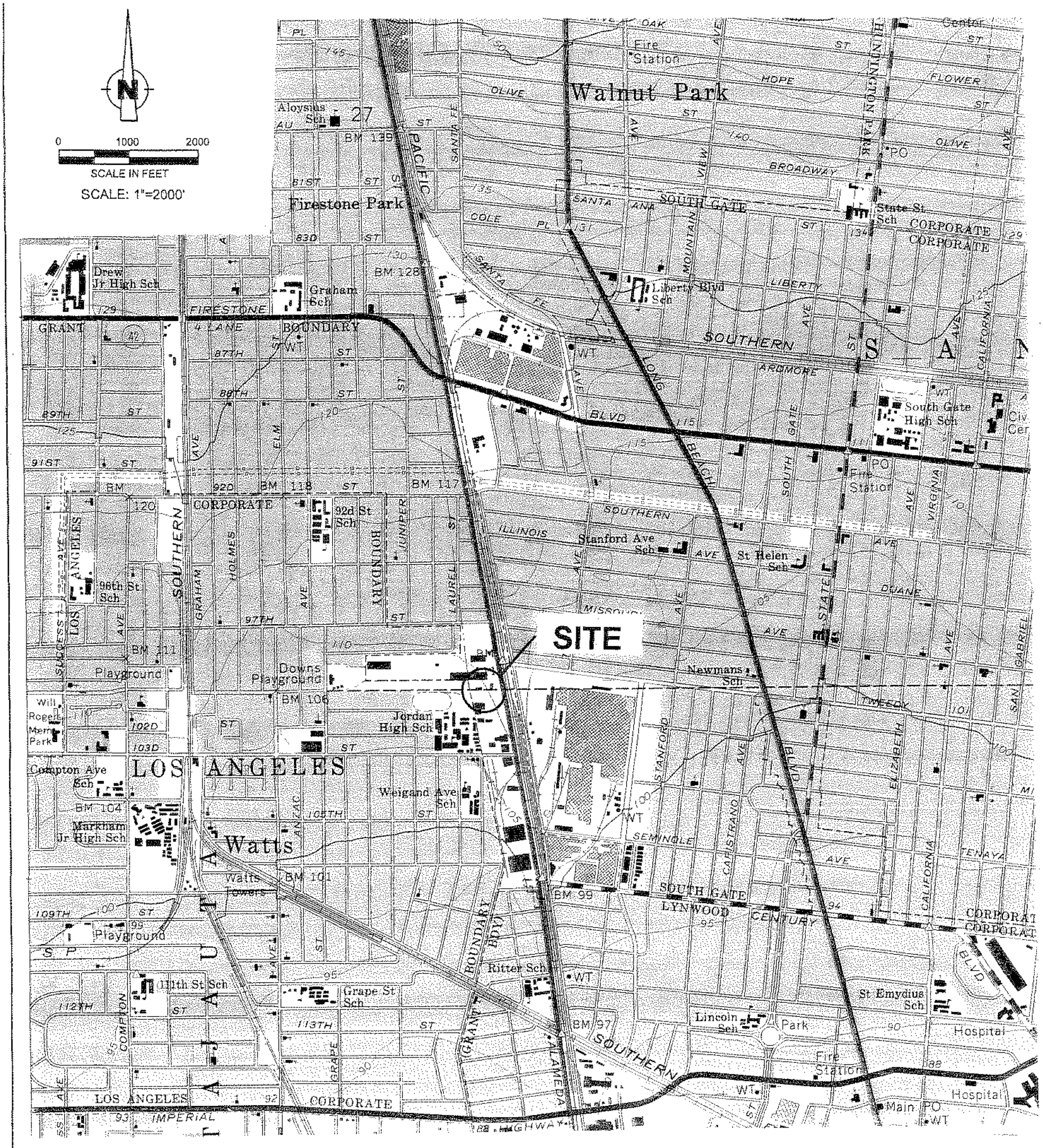


REFERENCE: USGS MAP
SOUTH GATE QUADRANGLE 1981

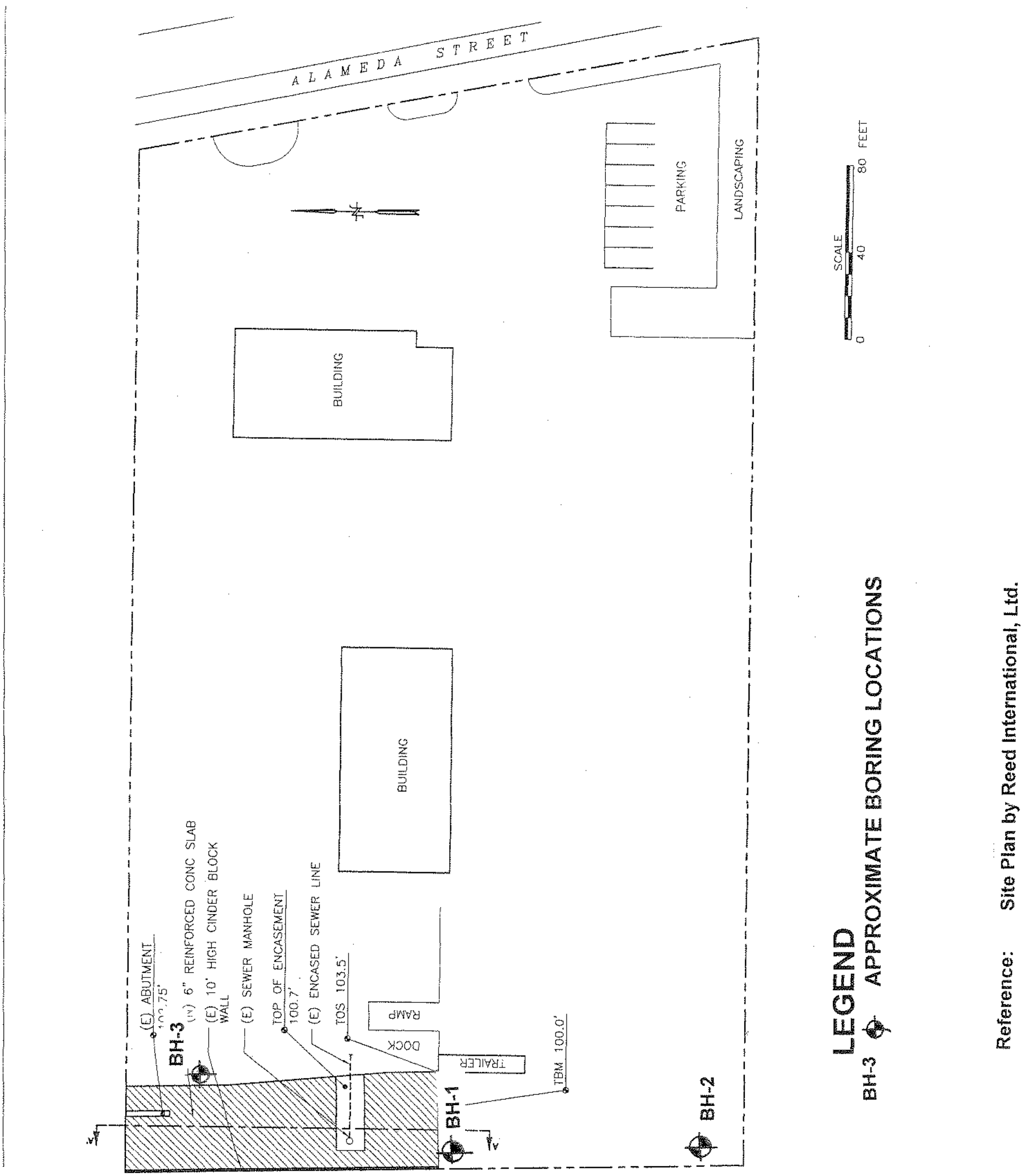
SITE LOCATION MAP

PROPERTY LINE WALL
AT ATLAS METALS
FOR SYSTEMS OPERATIONS SERVICES

Project No.
07-31-252-01

Converse Consultants

Figure No.
1



SITE PLAN AND APPROXIMATE BORING LOCATIONS PLAN

PROPERTY LINE WALL
AT ATLAS METALS
FOR SYSTEMS OPERATIONS SERVICES

Project No.
07-31-252-01

Converse Consultants

Figure No.
2

APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Field exploration included a site reconnaissance and subsurface drilling. During the site reconnaissance, surface conditions were noted, and the locations of the test borings were determined. Borings were approximately located using existing features as a guide.

Field exploration for the school expansion project consisted of drilling 3 exploratory borings (BH-1 through BH-3) to depths ranging from 2.5 to 51.5 feet below the existing ground surface at the locations shown on Figure No. 1, *"Site Location Map"*.

Test borings were advanced using a truck-mounted, 8-inch-diameter, hollow-stem auger drilling rig equipped for soil sampling. Soils were continuously logged and classified in the field by visual/manual examination, in accordance with the Unified Soil Classification System. Field descriptions have been modified, where appropriate, to reflect laboratory test results.

Boring No. BH-3 encountered refusal at a depth of 2.5 feet below the existing ground surface as a result of debris within the existing fill.

Relatively undisturbed samples of the subsurface soils were obtained at frequent intervals in the borings using a drive sampler (2.4-inch inside diameter, 3-inch outside diameter) lined with sample rings and a Standard Penetrometer Test (SPT) sampler. The steel sampler was driven into the bottom of the borehole with successive 30-inch drops of a 140-pound drive weight. An automatic ("safety") hammer was used. Blows required to drive the sampler six inches are shown on the boring logs in the "blows" column. Samples were retained in brass rings (2.4 inches in diameter, 1.0 inch in height) and carefully sealed in waterproof plastic containers for shipment to the Converse geotechnical laboratory. Standard Penetration Tests (SPT) were performed in general accordance with the ASTM Standard Test Method D1586-84. Blow counts given for each 6-inch increment are indicated on the boring logs, which is the uncorrected SPT *"N"*-value. Bulk samples of the near surface soils were also obtained.

Drawing No. A-1, *Exploration Log Key,* describes the various symbols and nomenclature shown on the logs. Logs of the borings are presented on Drawings Nos. A-2 through A-4, which also include descriptions of the soils encountered, pertinent field data, and supplemental laboratory results.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING LOG SYMBOLS

SAMPLE TYPE

STANDARD PENETRATION TEST
Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method

DRIVE SAMPLE 2.42" I.D. sampler.

DRIVE SAMPLE No recovery

BULK SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

LABORATORY TESTING ABBREVIATIONS

TEST TYPE
(Results shown in Appendix B)

CLASSIFICATION

Plasticity	pi
Grain Size Analysis	ma
Passing No. 200 Sieve	wa
Sand Equivalent	se
Expansion Index	ei
Compaction Curve	max
Hydrometer	h

STRENGTH

Pocket Penetrometer	p
Direct Shear	ds
Direct Shear (single point)	ds*
Unconfined Compression	uc
Triaxial Compression	tx
Vane Shear	vs
Consolidation	c
Collapse Test	col
Resistance (R) Value	r
Chemical Analysis	ca
Electrical Resistivity	er

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Project Name
PROPERTY LINE WALL
AT ATLAS METALS
FOR: SYSTEMS OPERATION SERVICES

Project No. 07-31-252-01

Drawing No. A-1

Log of Boring No. BH-1

Dates Drilled: 8/9/2007 Logged by: DA Checked By: JSS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		8" THICK CONCRETE SLAB						
		FILL (Af): **SILTY SAND (SM):** fine to medium-grained, scattered asphalt debris, black to dark brown.	■	▨	8/29	13	105	
5		**ALLUVIUM (Qal):** **SILTY SAND (SM):** fine-grained, brown.	■		10/6/6	15	98	ds,ma,h
			■		6/6/5	10	91	
10		**SANDY SILT (ML):** fine-grained sand, brown.	■		2/4/7	19	99	h
15		-dark brown	■		5/6/11	18	107	
		End of boring at 16.5 feet. Groundwater not encountered Boreholde backfilled and patched at the surface.						

Converse Consultants

Project Name
PROPERTY LINE WALL
AT ATLAS METALS
FOR: SYSTEMS OPERATION SERVICES

Project No. **07-31-252-01**

Drawing No. **A-2**

Log of Boring No. BH-2

Dates Drilled: 8/9/2007 Logged by: DA Checked By: JSS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): Depth to Water (ft): 43

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		8" THICK CONCRETE SLAB						
		ALLUVIUM (Qal):						max,ei,ca,er
		SANDY SILT (ML): brown.			4/5/7	18	104	
5		-less sandy, trace of clay			2/3/6	24	96	
		SILT (ML): trace of fine sand, dark brown.			3/2/4	24	99	ds,ma,h
10					2/4/4	15		
		SILTY SAND (SM): fine-grained, brown.						
15					3/5/8	17	100	
		SILT (ML): trace of fine sand, dark brown.						
20					1/2/2			
25		-slightly sandy, brown			2/3/4	22	102	
30		**SAND (SP):** medium-grained, tan.			5/8/12			
		CLAYEY SILT (ML): dark gray.						

Converse Consultants

Project Name
PROPERTY LINE WALL
AT ATLAS METALS
FOR: SYSTEMS OPERATION SERVICES

Project No. 07-31-252-01

Drawing No. A-3a

Log of Boring No. BH-2

Dates Drilled: 8/9/2007 Logged by: DA Checked By: JSS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): Depth to Water (ft): 43

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies *only at the location of the boring and at the time of drilling.* Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		CLAYEY SILT (ML): trace of fine sand.			1/3/4	31	89	
		SANDY SILT (ML): gray.						
40					5/14/18			
		SAND (SP): fine to medium-grained, gray.						
		SILT (ML): dark gray.						
45					6/8/12	16	107	
		SAND (SP): medium to coarse-grained, gray.						
50		**SILT (ML):** trace of fine sand, dark gray.			2/5/9			

End of boring at 51.5 feet.
Groundwater encountered at 43 feet.
Boreholde backfilled and patched at the surface.

Converse Consultants

Project Name
PROPERTY LINE WALL
AT ATLAS METALS
FOR: SYSTEMS OPERATION SERVICES

Project No. 07-31-252-01

Drawing No. A-3b

Log of Boring No. BH-3

Dates Drilled: 8/9/2007 Logged by: DA Checked By: JSS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		10" THICK CONCRETE SLAB (TOP OF LOADING DOCK)						
		FILL (Af): **SILTY SAND (SM):** fine to medium-grained, with abundant metallic debris, black.						
		Refusal at 2.5 feet due to large metallic debris. End of boring at 2.5 feet. Groundwater not encountered. Borehole backfilled and patched at the surface.						